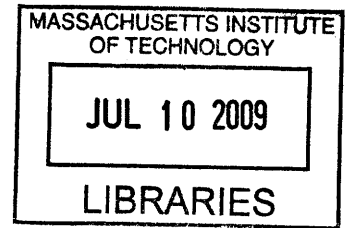


# The Design and Retrofit of Buildings for Resistance to Blast-Induced Progressive Collapse

by

Isabel Abbott Galvão Sobreira Lopes

B.Eng. Civil and Environmental Engineering  
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Signature of the author

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Department of Civil & Environmental Engineering  
May 18, 2009

Certified by

---

Jerome J. Connor  
Professor of Civil & Environmental Engineering  
Thesis Supervisor

Accepted by

---

Daniele Veneziano  
Chairman, Departmental Committee for Graduate Students

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Isabel Abbott Galvão Sobreira Lopes

Submitted to the Department of Civil and Environmental Engineering on May 18, 2009 in Partial Fulfillment of the Requirements for the Degree of Master of Engineering in Civil and Environmental Engineering

## ABSTRACT

In recent years, concern has risen drastically regarding the suitability of structural design for blast resistance. Historic events have proven that buildings that are designed in compliance with conventional building codes are not necessarily able to resist extreme loads, particularly blast loads. While significant progress has been made towards the development of design guidelines for federal buildings according to threat level, comprehensive codes have not yet been devised for blast hardening of non-federal buildings.

One of the most important considerations in the strength-based design of buildings is adequate protection against progressive collapse in any unforeseen event. In the past, this has been achieved to a limited extent through exterior barriers, which help thwart efforts to directly impact the building, and thus minimize damage. With the emergence of structural design for blast resistance, methodologies to inhibit progressive collapse can expand to not only comprise exterior components, but also the structural components themselves.

This thesis outlines potential structural and architectural techniques to design and retrofit buildings for resistance to blast impact loads as well as progressive collapse. These considerations are then applied to the study of two well-known blast events, the 1995 bombing of the Alfred P. Murrah Building in Oklahoma, and the 2001 attack on the Pentagon. A comparison of the two cases reveals that the redundant and ductile design of the Pentagon provided considerable resistance to impact loads, and contributed greatly to impeding the onset of progressive collapse. Several blast-resistant features found in the Pentagon were not present in the design of the Murrah Building, thereby increasing the vulnerability of the structure to damage and collapse.

The development of design codes and guidelines for blast resistance presents a number of challenges. These challenges generally arise from the erratic nature of blast loads and the difficulty in standardizing design procedures for variable levels of threat. While it may be difficult to implement general guidelines and codes for blast resistance, existing knowledge of blast hardening techniques can be applied to the design of buildings on a risk-based, case-by-case basis.

Thesis Supervisor: Jerome J. Connor

Title: Professor of Civil and Environmental Engineering

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I would like to dedicate this thesis to my grandfather, Fernando Abbott Galvão, who shaped the family that we are today through his unconditional love and his incessant determination to fulfill our dreams. He will be greatly missed and forever revered.

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## 1. INTRODUCTION

Extreme blast events in recent history have prompted a re-evaluation of existing design methods and codes to consider the effects of blast loads on structures and their occupants. Several cases of terrorist attacks on government buildings have led to research and recommendations for more robust and ductile design, particularly for highly targeted buildings. Aside from hostile bomb blasts, certain buildings are also more susceptible to accidental blasts as a result of the functions they perform. For instance, facilities that manufacture or handle flammable or explosive materials are inherently at greater risk of accidental explosion events. This thesis addresses the effects of blasts on buildings and presents various design solutions that can potentially mitigate these effects.

Chapter 2 contains a basic description of bomb blasts and the detrimental properties that the ensuing shockwaves may have on adjacent structures. The damage inflicted by a blast comprises both immediate effects associated with the impact of the shockwave and debris on structural elements, and the possible collapse of the building or part of the building due to the loss of supports. The ideal design of buildings for blast resistance takes both of these possible consequences into consideration, as described in detail in Chapter 3.

Two of the most important aspects of blast resistance in buildings are structural redundancy and ductility. In addition to such structural considerations, the architectural layout and exterior components of buildings can also be optimized to either prevent or mitigate blast effects. Chapter 3 presents the significance of designing for redundancy and ductility for the purpose of providing resistance to blast impact loads as well as subsequent progressive collapse. It further describes examples of structural and architectural features that can be implemented to provide such resistance.

Existing buildings are often re-evaluated to assess their resistance to new or altered loads. The hardening of existing buildings to more effectively resist blast loads has become a key element of retrofit considerations, particularly in older government buildings that were not necessarily designed for high blast threats. Chapter 4 describes a few procedures for retrofit, including non-intrusive elements, such as exterior moment frames, and interior elements, such as additional shear walls.

Chapter 5 applies the aforementioned considerations to a comparison of two case studies. The first, a study of the partial progressive collapse of the Alfred P. Murrah Building in Oklahoma City, addresses some structural and architectural weaknesses that caused the building to be more prone to impact damage and progressive collapse following a terrorist bomb explosion. A few retrofit strategies are then presented as a hypothetical means to provide blast hardening to the Murrah Building under the given blast load scenario. The second case review presents the results of the Pentagon Building Performance Study (BPS) following the September 11 attack. It describes the conclusions of the BPS team as to the properties of the Pentagon that enhanced its resistance to progressive collapse.

Finally, Chapter 6 addresses the difficulties involved in the implementation of guidelines and codes for blast resistant design. There are a number of challenges associated with the assessment of blast threat levels as well as the erratic nature of blast loading, which inhibit the development of standard procedures for design. While it may not be possible to incorporate blast resistance into building codes based on existing knowledge of blast loads, certain design techniques have the potential to mitigate the effects of blasts on a case-by-case basis, depending on the results of discrete risk analyses.

## **2. BLAST CHARACTERISTICS AND POTENTIAL DAMAGING EFFECTS**

### **2.1 BLAST TYPES AND THEIR CHARACTERISTICS**

In analyzing the effects of blasts on structures, the properties of any given explosive are conventionally adjusted relative to the properties of TNT, such that they can be studied in a standardized manner. The immediate pressure created by the blast is known as the peak pressure. The explosion of TNT results in a shockwave which, as long as its path is unobstructed, loses pressure as it travels away from the initial blast. However, if the shockwave comes into contact with an object, it is reflected and amplified by the object. Depending on the intensity of the blast and the angle of incidence of the wave, the pressure of the reflected shockwave can be several times larger than the peak pressure (Punch, 1999).

Blast loads are among the most variable and unpredictable loads to which a structure can be subjected. The highly impulsive effects of blasts have the potential to cause severe destruction in a very short period of time. These characteristics give rise to difficulties in design, not only for immediate impact but for the overall resistance of the structure to progressive collapse.

### **2.2 IMMEDIATE DAMAGE**

The most crucial aspect of blast events in relation to structural behavior is the immediate response of the structure to the shockwave produced by the blast and to the ensuing impact of debris. The state in which the structure emerges from a blast dictates the degree of load redistribution that will occur and the capacity of the structure to accommodate such changes in load path without collapsing.

Explosions generate shockwaves which, depending on the intensity of the blast, travel through concrete at 2,700-3,400 m/s and through steel at 4,900-5,800 m/s. These shockwaves produce strains in the material at very high rates, thus leading to fragmentation and disintegration, or *brisance*. The direct contact of the explosive with the structure produces primary fragments, which then create secondary fragments as they are propelled against other components of the building. The degree of damage resulting from a blast varies with the distance to the location of the explosion, which is directly related to the amount of pressure exerted by the explosion. This relationship can be quantified by a variable known as the scaled range (NRC, 1995):

$$Z = \frac{R}{W^{1/3}}$$

Where

R is the radial distance to the location of the blast; and

W is the weight of explosive, adjusted to an equivalent weight of TNT.

Under this assumed relationship, for any single value of Z, the blast effects (such as peak pressure and duration) are constant. Thus, the scaled range provides a useful means to model the effects of a blast on different structural elements, by varying one parameter at a time.

At a large enough scaled range (which is a result of either a small blast pressure or a large distance from the blast), the blast has a global dynamic effect and is distributed among all elements of the structure. At a smaller scaled range, building elements that are adjacent to the explosion (most commonly exterior walls and façades) are likely to undergo disintegration upon impact, whereby fragments spall off or are propelled towards the interior of the building. As the shockwave and debris progress further away from the blast location, extensive cracking may occur without complete

disintegration. In some instances, a member may experience partial disintegration of the near face and spalling of the far face (NRC, 1995).

## **2.3 PROGRESSIVE COLLAPSE**

Progressive collapse is a term that is generally used to describe the failure of a structure or part of a structure to redistribute gravity loads, often following the loss of one or more supports or transfer elements. In order to evaluate the resistance of a building to progressive collapse, analysis procedures generally assume a doubled gravity load to account for the sudden loss of structural elements. For this purpose, it must be ensured that the capacity-to-demand (C/D) ratio is greater than 2 for both flexure and shear (Corley, 2008).

In 1968, a residential building in England, called Ronan Point, experienced partial progressive collapse following an accidental gas explosion. The tragedy prompted the formulation of a set of clauses in the British design code to address progressive collapse. These clauses were later compiled into a document named Document A 2004 A3 *Disproportionate Collapse*. The document stipulates that if the hypothetical removal of any of the following elements causes progressive collapse, then that element is a key supporting element and must be designed for blast loading (Gurley, 2008).

- Any column;
- Any transfer beam supporting two or more columns;
- Any span of load-bearing wall between two lateral supports, not exceeding 2.25H long (where H is the story height).

By incorporating these considerations into modelling and analysis of buildings, structural engineers can more effectively gauge the resistance of buildings to progressive collapse, and thereby modify the structural elements and load paths accordingly.

### **3. DESIGN CONSIDERATIONS FOR BLAST RESISTANCE**

Emerging concerns regarding terrorist threats have resulted in a growing tendency to design buildings for blast resistance. Blast-hardening is a term commonly used to refer to any design or retrofit method for the mitigation or elimination of blast effects on a structure (NRC, 1995).

#### **3.1 REDUNDANCY**

The overlying design characteristic that prevents progressive collapse is structural redundancy. This can be achieved by increasing the number of members (thereby reducing beam spans and column spacing) or the size of the members, or by enhancing the detailing of reinforcement. Structural redundancy implies that the structure has more than one alternative load path for the transfer of gravity loads. Thus, in the event of the loss of a column or girder, the redundant elements will effectively carry the redistributed load.

The National Science Foundation defines the three key components of structural redundancy as follows (Husain and Tsopelas, 2004):

- i. The level of static indeterminacy;
- ii. The ratio of the probability of global failure to the probability of failure of any single member;
- iii. The level of capacity beyond that required for safe design.

Following recent terrorism events in the United States, the General Services Administration (GSA) has established a requirement that all new government buildings be designed for the loss of a perimeter column on the first and second floors without inducing progressive collapse. In order to conform to this and other redundancy specifications, a number of structural developments can be considered.



### 3.1.1 CONTINUOUS REINFORCEMENT THROUGH SUPPORTS

One technique by which redundancy can be achieved in concrete structures without reducing any usable space is the design of reinforcement to account for atypical loads or possible changes in load path. Column loss is a plausible event when a blast occurs at close proximity to a building. In order to mitigate this risk, or to prevent further collapse in the case of a lost column, the longitudinal reinforcement of the girders can be designed continuously through the columns that support them. Typically, reinforced concrete beams and girders are designed such that the longitudinal reinforcement is curtailed only a short distance into the support. This is allowable because the column or girder that supports it is able to carry the flexural load at that location. However, if the supporting member suddenly loses structural capacity, the beam or girder above must span a greater distance, and therefore must resist a much higher load in flexure. If the positive longitudinal reinforcement is designed to continue through the support, the flexural capacity of the member at the support will be equal to the capacity at the interior; therefore, the member will be more likely to support itself in flexure.

### 3.1.2 NEGATIVE REINFORCEMENT

The reinforcement detailing of concrete slabs can also be designed for redundancy, particularly to account for the possibility of unforeseen uplift pressures that may result from a bomb blast. Floor slabs are generally designed for positive moments at mid-span, which are produced by gravity loads. Thus, the predominant longitudinal reinforcement is located on the bottom of the slab. In the event of an explosion under the floor slab, a large uplift pressure will emanate from the blast and induce a large negative moment in the slab. More robust design of the top reinforcement would more effectively counterbalance such blast-induced uplift loads.

### 3.1.3 TWO-WAY FLOOR SYSTEMS AND SHORT SPANS

The design of floor systems to span in two directions increases the possibility for load redistribution in the event of member loss. In such a system, secondary beams can be designed to span perpendicularly to the main girders. This technique not only reduces floor spans, but it also provides alternative load paths for the prevention of progressive collapse. An example of an efficient two-way floor system is presented in Section 5.2 The Pentagon, which addresses the positive design aspects that aided in the resistance of the Pentagon to progressive collapse following the September 11 attack.

One of the most challenging aspects of incorporating structural redundancy into a design code for blast resistance is the need to establish a compromise between efficiency and safety. Existing codes use safety factors with the objective of designing for sufficient levels of safety, without exceeding the necessary amount of material and labor. However, when blast resistance is taken into consideration, it becomes more difficult to determine what level of redundancy will be sufficient to forestall the onset of progressive collapse.

## 3.2 DUCTILITY

Ductility is critical to the performance of a building under blast loading. Ductile design prevents members and connections from failing in a sudden, explosive manner, as it increases their capacity to absorb energy through deformation. In a blast event, the capacity of beams, columns, and slabs to undergo considerable deformation may not only prevent the loss of structural elements, but it may also contribute to the capacity of the structure to accommodate redistributed loads.

### 3.2.1 COLUMN DESIGN

One important consideration for the design of reinforced concrete buildings for increased ductility is the nature and distribution of transverse reinforcement of columns. Reinforced concrete columns are typically designed for transverse shear resistance by means of either ties or spirals. In addition to providing shear resistance, this form of reinforcement also serves to confine the longitudinal reinforcing bars within the concrete. While both types of reinforcement (ties and spirals) can be adequately designed for a specified shear resistance, spiral reinforcement provides a higher degree of confinement to the concrete core, thereby inhibiting the sudden buckling of the longitudinal reinforcement. Buckling is a brittle form of failure that can be observed in columns, particularly under combined axial and lateral loading, such as in a blast event. Thus, the mitigation of buckling is a critical aspect of design for blast resistance.

In steel buildings, a comparable increase in toughness and ductility can be obtained by designing concrete-filled tube columns along the perimeter. This technique is a cost efficient and easily constructible alternative to wide flange moment frame columns, which would require a much greater weight to achieve the same toughness (Punch, 1999).

### 3.2.2 CONNECTION DESIGN

Section 3.1 Redundancy describes methods to design beams and columns for increased redundancy. While this aspect of design is crucial for blast resistance, redundancy is not effective unless the connections are also fit to safely redistribute loads. For this purpose, connections must be designed such that they will undergo large inelastic deformation and rotation due to overload from the plastically deformed beam or girder. In steel, a type of connection called SidePlate has been developed as an effective moment connection, making use of several important attributes for

enhanced ductility, continuity, and constructability. These attributes are described in detail and illustrated below (Punch, 1999).

### 1. Ductility

The vertical plates are welded to either side of the column in the direction parallel to the shear force, as shown in Figure 1. The orientation of the welds parallel to the force provides excellent ductility, as the welds can undergo considerable plastic deformation.

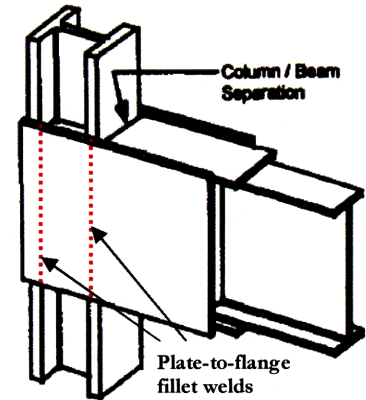


Figure 1: Ductility of the SidePlate Connection  
(from Punch, 1999)

### 2. Continuity

The vertical plates (highlighted in yellow in Figure 2) provide a connection between the two beams spanning into the column. If that column is lost, the two beams can remain connected and thus span the distance between the outer pair of columns as a single continuous beam.

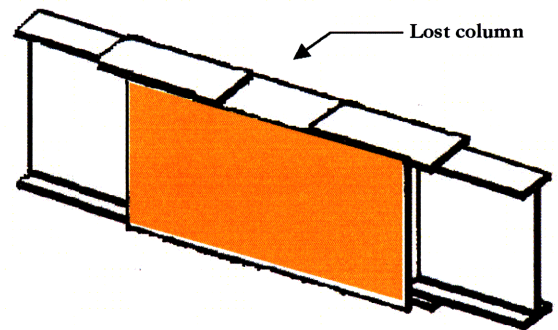


Figure 2: Continuity of the SidePlate Connection  
(from Punch, 1999)

### 3. Constructability

With the exception of the plate-to-flange fillet welds depicted in Figure 1, the SidePlate connection is fabricated and welded to the beam ends offsite. Once the elements are transported onto the site, they are easily connected by the fillet welds shown. The ease of

assembly of frame members using SidePlate connections considerably reduces the time and labor required during construction.

### **3.3 ARCHITECTURAL FEATURES**

When considering methods to mitigate the effect of blasts on structures, the primary concern tends to revolve around the design of the structural components to resist impact loads and inhibit progressive collapse. While structural hardening undoubtedly reduces the vulnerability of a building to blast effects, the most effective and feasible technique to manage the risks associated with blasts is to design the architectural features of the building and its surroundings such that it is more difficult for explosions to occur in or near the building. In addition, certain building components can be optimized for safe and efficient egress, and emergency plans can be established to permit such evacuation procedures.

#### **3.3.1 FAÇADE DESIGN**

One of the most important architectural components of a building for blast resistance is the exterior façade. Ideally, when a window is subjected to lateral pressure, it should not shatter in a brittle manner; rather, failure should occur in a controlled manner such that the fragments are well contained. The adequate design of windows using high-strength glazing and other protective features can not only contribute to the resistance of the structure to the initial impact of an explosion, but it can also control the vulnerability of its occupants to impact from window fragments and other projectiles (NRC, 1995). Furthermore, as will be addressed in Section 5.2 The Pentagon, the exterior walls can be constructed with brick infill, which in the event of a blast may aid in the redistribution of gravity loads due to lost supports.

### 3.3.2 ACCESSIBILITY

Aside from controlling the ingress of explosives and projectiles through the exterior walls, additional obstacles can be implemented to prevent access of external threats to the vicinity of the building. This can be done by constructing traffic barriers around the perimeter of the building in order to restrict the entrance of unauthorized vehicles beyond a certain point. Strict controls can also be placed on access of external vehicles to underground parking and loading/unloading areas, thereby mitigating the dangers associated with upward blast pressures. The architectural layout of the building itself can also be planned according to the vulnerability of its occupants to external threats. For instance, highly populated areas such as offices and day care facilities can be located in more remote parts of the building, while hallways and storage rooms can be located closer to public traffic (NRC, 1995).

### 3.3.3 CONTROL OF INTERNAL EXPLOSIONS

Depending on the function of a building, its susceptibility to internal explosions may prevail over external (often terrorist-related) dangers. In order to account for the possibility of accidental internal blasts, the design of buildings that are intended for fabrication or handling of explosive materials should include mechanisms for control and release of blast pressure. One such mechanism, known as a blast valve, can provide gradual release of pressure to the exterior of the building in the case of a sudden increase in internal pressure. Blow-out panels and other redundant elements can also be designed to undergo failure prior to other building components, with the purpose of alleviating pressure following an explosion. These elements help prevent the amplification of shockwave pressures that would otherwise originate due to reflection from rigid objects (NRC, 1995).

### 3.3.4 EMERGENCY PROCEDURES

Finally, adequate design and planning for an emergency event such as a blast can allow for the safe egress of building occupants and avoid the threat of imminent collapse or fire-related hazards. Stair shafts can be designed with heavy fireproofing and resistance to smoke penetration, in order to provide a safe means of refuge and evacuation in the event of a fire. Additional areas of the building can also be designed specifically for refuge, using pressurization or compartmentation to prevent smoke penetration, while occupants are not able to exit the premises of the building (NRC, 1995). Regardless of blast resistance considerations used in design, the architectural and structural features of a building do not necessarily suffice to guarantee the safety of its occupants in the event of a blast. In order to account for this uncertainty, the provision of a comprehensive emergency plan is critical to the effective design of a blast resistant structure.

## **4. RETROFIT OF EXISTING BUILDINGS FOR BLAST RESISTANCE**

Oftentimes, existing buildings are retrofitted to adapt to changes in loading. Older buildings may not have been designed in consideration of blast resistance to as great an extent as new buildings (especially federal buildings) are presently designed. Building owners may therefore opt to harden the structure in response to new threats. Blast hardening can be achieved in existing buildings in several ways, both on the interior and the exterior of the building.

### **4.1 EXTERIOR MOMENT FRAMES**

Exterior moment frames are an effective means to add robustness to a structure without altering existing components. An application of exterior moment frames is presented in Section 5.1 The Alfred P. Murrah Building for the hypothetical retrofit of the Alfred P. Murrah Building in Oklahoma City, which was partially destroyed by a terrorist bomb blast. Exterior moment frames provide resistance against the immediate effects of a blast by acting as sacrificial elements, thereby inhibiting the destruction of the interior structure due to impact. Furthermore, if some loss is still sustained by the building, the moment frame provides redundant load paths for the redistribution of vertical loads, and thus helps prevent progressive collapse.

### **4.2 ADDITIONAL STRUCTURAL WALLS**

The retrofit of buildings for blast hardening can also involve the addition of structural walls to either the exterior or the interior of the existing structure. Additional structural walls can serve to enhance resistance to lateral loads, such as the shockwave induced by a blast, as well as to increase the overall robustness and redundancy of the structure so as to inhibit the onset of progressive collapse following structural loss. Section 5.1 The Alfred P. Murrah Building also addresses the use of



additional exterior and interior structural walls, to evaluate the hypothetical effectiveness of retrofitting the Alfred P. Murrah Building for enhanced blast resistance.

## 4.2 COLUMN JACKETING

Column jacketing is an emerging technology that has been applied to existing reinforced concrete columns for a variety of structural purposes, often related to seismic resistance. While the more conventional method to apply column jacketing is by reinforcing the exterior of the column with a steel jacket, new material developments have created a trend towards fiber reinforced polymers and other composite materials for their ease of installation and aesthetic qualities.

Crawford et al. (2001) describe a means to reinforce columns in both shear and flexure by applying carbon, glass, or Kevlar fiber reinforced polymers in various forms, for the purpose of enhancing the resistance of RC columns to blast. Figure 3 illustrates an example of a retrofitted square column for shear and flexural resistance to blast loading. While the longitudinal strips provide additional flexural capacity, the transverse hoop wrap enhances the shear capacity of the column.

In order to assess the need for column jacketing, Crawford et al. developed a procedure whereby the peak

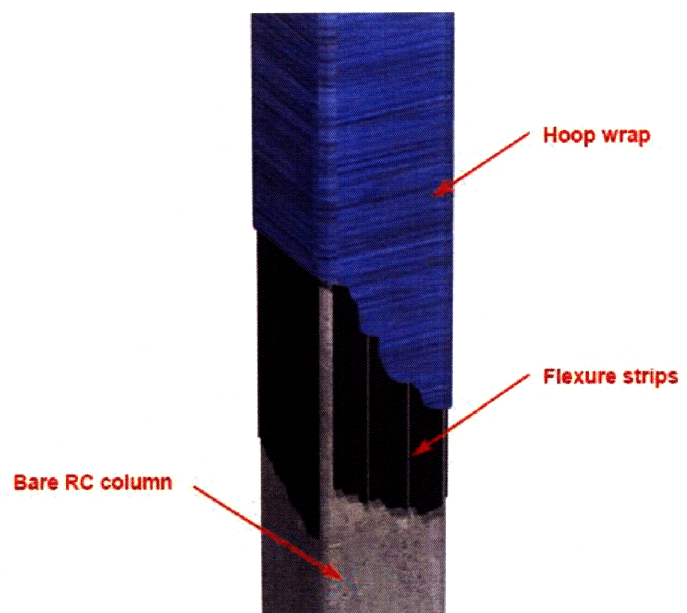


Figure 3: FRP Column Jacketing (Crawford et al., 2001)

displacement of a modeled column under a hypothetical blast load is tested against the maximum allowable displacement. If the observed displacement is found to be unacceptable, FRP strips and hoops are designed with adequate thicknesses to overcome the excessive displacement.

## 5. CASE STUDIES

### 5.1 THE ALFRED P. MURRAH BUILDING

The partial collapse of the Alfred P. Murrah building in Oklahoma City is one of the most well-known cases of blast-induced progressive collapse in the United States. The incident occurred in April 1995, following a terrorist truck bomb attack. 168 lives were lost as a result of the immediate blast effects as well as the progressive collapse that ensued (Corley, 2008).

#### 5.1.1 STRUCTURAL SYSTEM

The Murrah building was a 9-story reinforced concrete federal office building, with additional 1-story east and west wing buildings. Figure 4: below depicts the main structural features of the

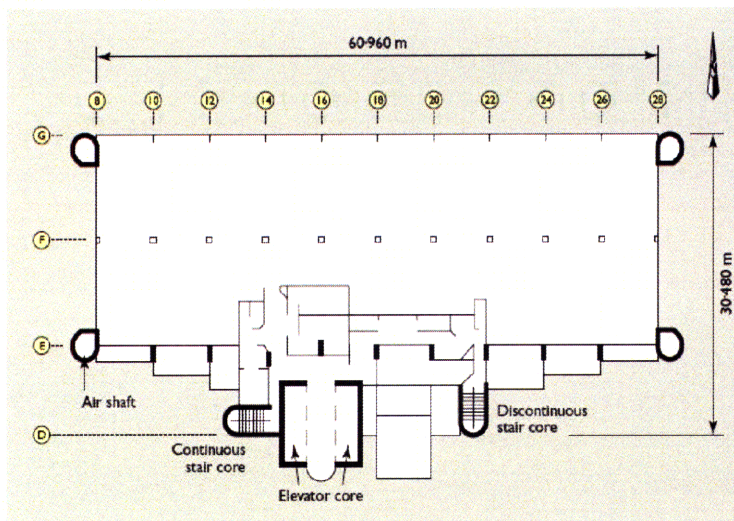


Figure 5: Reinforced Concrete Plan of the Alfred Murrah Building (Corley, 2008)

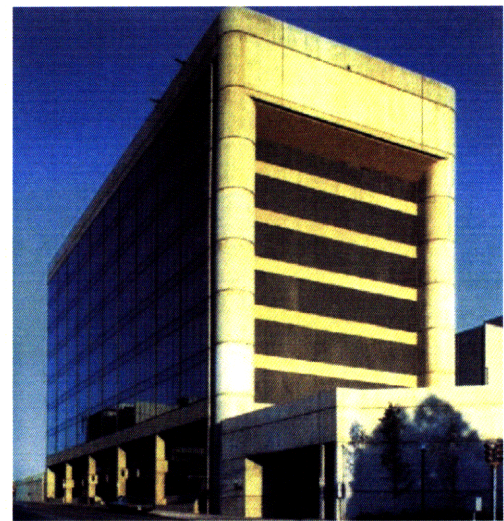


Figure 4: The Alfred Murrah Building (Corley, 2008)

building, noting the orientation of the column grid and the locations of the shear walls. It is evident from Figure 4 and Figure 4: that the lateral load resisting elements were concentrated in the back (south) of the building, leaving a more open configuration along the front (north) façade, where the main entrance was located. This was achieved with the aid of transfer girders at the 3<sup>rd</sup> story level,

which spanned twice the distance of the typical upper-story beams and carried the vertical loads down to the columns at every other gridline. Due to the longer spans and higher loads on these columns, their structural integrity following a harmful event would have been crucial to the overall resistance of the building to progressive collapse.

### 5.1.2 ACCIDENT INVESTIGATION

Following the disaster, the American Society of Civil Engineers (ASCE) and the Federal Emergency Management Agency (FEMA) conducted detailed investigations into the progression of events from the initial blast to the partial collapse of the building. These studies were concluded with the indication of a number of structural weaknesses in the Murrah building, which may have intensified the level of damage to the individual components and helped trigger progressive collapse (Corley, 2008).

The blast occurred at gridline 20, near the center of the north façade, as represented by the bomb crater in Figure 6. Due to its close proximity to column G20, the direct effect of the blast was the

immediate removal of that column by brisance. The transfer girder was thus forced to span 24 m, between columns G16 and G24. The blast also caused approximately 10% of the floor area to fail due to uplift

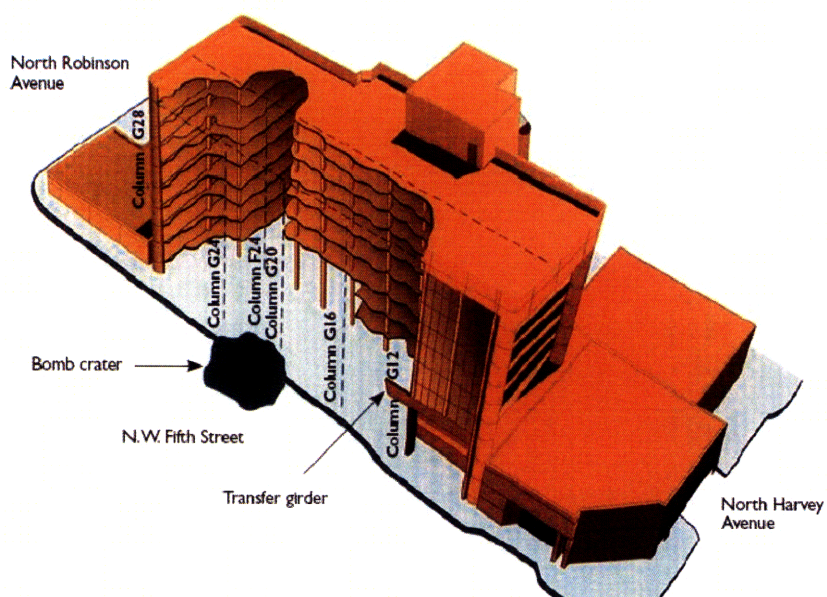


Figure 6: Collapsed Alfred Murrah Building (Corley, 2008)

pressures in excess of the slab self-weight (Gurley, 2008).

The most destructive outcomes of the bomb blast were not caused directly by the blast pressure, but indirectly by the detrimental effects of the blast on the load-carrying capacity of the structure. The studies done by ASCE and FEMA concluded that the loss of column G20, and thereby the doubled span of the transfer girder between columns G16 and G24, drastically increased the moments at the edge supports of the girder. This also increased the shear at the supports, which was not sufficiently resisted by columns G16 and G24. Consequently, these two columns failed in shear and doubled the span once again, from 24 m to 48 m. The inability of the transfer girder to span such a length led to the progressive collapse of the north half of the building, all of which was directly or indirectly supported by columns G16 and G24 (Gurley, 2008). In the absence of three columns along gridline G, the lack of an alternate load path rendered the building incapable of transferring gravity loads through the remaining columns.

### 5.1.3 CAUSES OF FAILURE

The FEMA/ASCE report attributed the failure of columns G16 and G24, as well as the transfer girder and the floor slabs, to detailing of the reinforcement. In the case of the columns, the spacing of the transverse reinforcement and the connection detail between the columns and the girder were in compliance with the requirements for an ordinary moment frame, but did not provide sufficient confinement or shear resistance to the additional load induced by the doubled span of the girder. The girder, in turn, was not designed to resist the magnified tension at midspan caused by the removal of columns G16 and G24. Since the transfer girder had been originally designed to span 12 m between columns, the bottom reinforcement at these locations was not adequate in resisting such large tensile stresses along the bottom face. Similarly, the reinforcement of the floor slabs had been designed predominantly for gravity loads, and hence for positive moments at midspan. Thus, the top



reinforcement was not effective in resisting the negative moment produced by the uplift pressure of the bomb blast (Gurley, 2008).

In addition to deficiencies related to the reinforcement detailing of the concrete structure, the response of the building to the bomb blast and its after-effects was also influenced by the asymmetrical distribution of shear walls with respect to the center of mass of the building. The concentration of shear walls near the south face of the building increased its susceptibility to global torsion effects from lateral loads (Corley, 2008).

#### 5.1.4 HYPOTHETICAL RETROFIT SCHEMES

As a supplement to the ASCE/FEMA report, Gene Corley (2008) presented three possible schemes by which the Murrah building could have been retrofitted to prevent progressive collapse from a blast similar to that of the 1995 terrorist attack. All three schemes were founded on FEMA's proposal to design for blast resistance on the basis of seismic design, according to ACI 318.

The three proposed alternatives for seismic retrofit are presented below, along with the intended function of each added structural component in providing resistance to immediate blast effects and progressive collapse.

##### 1. External Piers and Spandrels

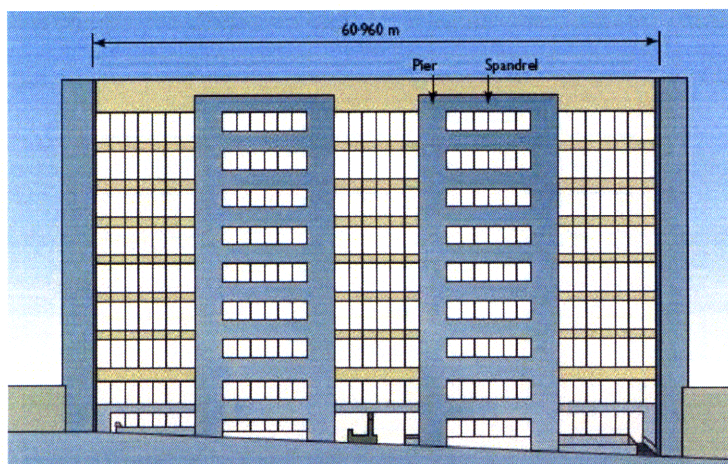


Figure 7: External Piers and Spandrels (Corley, 2008)

Figure 7 presents the pier and spandrel scheme, which consists of two sets of rigidly-connected piers and spandrels, arranged symmetrically along the north

façade. The large horizontal openings were designed so as to make optimal use of the existing window openings. The two additional walls are connected to the original columns and girders along gridline G by means of dowels and are founded on existing column foundations, in order to provide desired interaction for seismic resistance (Corley, 2008).

## 2. External Special Moment Frame

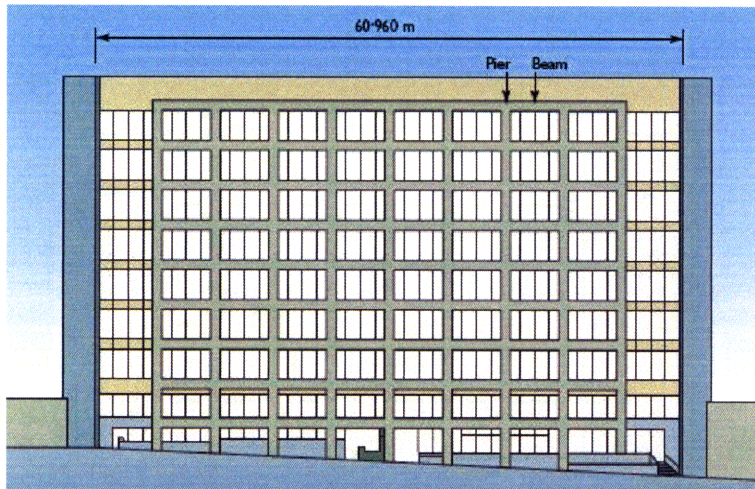


Figure 8: External Moment Frame (Corley, 2008)

Under the special moment frame scheme, an additional reinforced concrete special moment frame is constructed along gridline G, between columns G10 and G26, as shown in Figure 8. All girders were designed with the same depth, with the exception of the

top two stories, which do not have to transfer as much load. Similarly to the pier and spandrel scheme, interaction is established between the original structure and the special moment frame by dowels and shared foundations (Corley, 2008).

## 3. Additional Internal Shear Walls

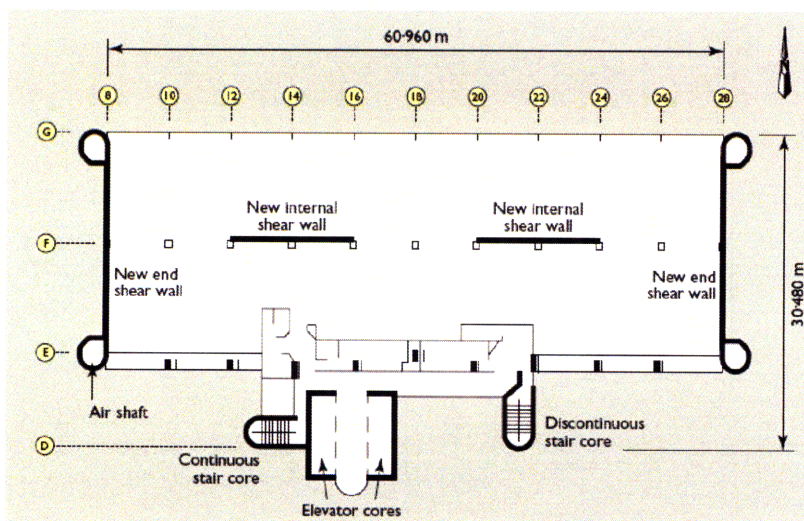


Figure 9: Additional Shear Walls (Corley, 2008)

The third alternative, depicted in Figure 9, consists of the addition of two internal shear walls, one between columns F14 and F16 and the other between

columns F20 and F24, as well as two external shear walls along the short (east and west) ends of the building. This option allows for retrofit while preserving the original appearance of the north façade (Corley, 2008). Although the shear walls do not enhance the robustness of the structure at the location of the blast, their symmetrical orientation adds to the torsional stiffness of the building. Furthermore, the new shear walls provide structural redundancy, and hence diminish the susceptibility of the building to progressive collapse. The wall ends were also designed with heavier reinforcement for effective post-yield seismic response (Corley, 2008).

#### 5.1.5 ANALYSIS RESULTS

Progressive collapse analyses reported by Corley revealed that the two external schemes reduced the total collapsed area of the building from 42% to 4% of the original area. The internal shear wall scheme also lowered the extent of collapse, but only to approximately 31-34% of the original floor area.

The pier and spandrel system effectively limited column loss to pier G20, which would fail between the first and second stories as a direct consequence of the blast. In addition, the spandrel beam between columns G20 and G24 would be partially shattered. The structure was then analyzed to evaluate whether the remaining piers and spandrels had the capacity to support the redistributed load due to the lost pier. 2D and 3D models revealed a capacity-to-demand ratio of about 2, implying that the frame had sufficient capacity to prevent any further progressive collapse. It was assumed that the floor slabs that had originally failed as a direct result of the blast-induced uplift pressure would still fail under the new scheme (Corley, 2008).

In the models of the special moment frame system, it was observed that column G20 was lost from the 1<sup>st</sup> to the 3<sup>rd</sup> story and column G22 was lost at ground level, both due to shattering from



immediate blast effects. These immediate effects, combined with the loss of support from columns G20 and G22, caused the beam on the 2<sup>nd</sup> story to fail between columns G18 and G24 as well. Furthermore, it was found that the beams on the roof and the 9<sup>th</sup> floor yielded beyond their yield capacity. Despite this, the beams on the lower floors had enough capacity to resist this redistributed load in flexure. With a capacity-to-demand ratio of 2.3, the special moment frame would not undergo progressive collapse. Therefore, similarly to the pier and spandrel system, the only floor area that would be lost in the special moment frame scheme would be the area directly affected by the bomb blast (Corley, 2008).

The improved response of the building to the simulated bomb blast under the two external strengthening schemes was attributed to enhanced shear and tension strength as well as increased toughness. The use of thicker beam and column cross-sections and greater longitudinal and transverse reinforcement provided increased strength to the building components. Furthermore, greater continuity of longitudinal reinforcement provided a higher capacity to absorb energy, by effectively redistributing loads resulting from lost supports. The retrofitted building was also more resistant to direct blast loads through the use of more robust members.

The Alfred Murrah building has been a valuable case study in the field of blast resistance, as it has led to numerous investigations of potential techniques for the improvement of structural response to blasts. Existing information concerning the actual response of the building to the 1995 terrorist attack has permitted more realistic simulations and thus more accurate predictions of structural behavior under blast loads.

## **5.2 THE PENTAGON**

Studies of the performance of the Pentagon in response to the September 11 attack have also contributed greatly to the development of blast resistant design techniques. Investigative reports

have praised the structural design of the Pentagon for its exemplary resistance to progressive collapse following a very large impact from a Boeing 757 jet.

The Pentagon is a massive 5-story reinforced concrete building that serves as the headquarters of the U.S. Department of Defense, in Washington, D.C. The structure was undergoing a major renovation when it was struck by a hijacked airliner on September 11, 2001. The impact and its after-effects caused significant damage to part of the building and killed 189 people. Almost immediately after the attack, the ASCE assembled a building performance study (BPS) team to investigate the response of the structure and thereby propose recommendations for future design (Mlakar et al., 2003).

The impact of the aircraft occurred in a portion of the building that had already been renovated. The renovation had consisted of the installation of new elevators, stairwells, escalators, and mechanical equipment rooms, in addition to the retrofit of the exterior walls and windows for enhanced resistance to high pressures. In spite of the considerable destruction that resulted from extreme impact loads, the structure demonstrated exceptional resistance to progressive collapse and further damage, predominantly due to structural redundancy, continuous beam reinforcement through supports, spiral reinforcement of columns, and the capacity of the exterior walls to behave as transfer girders (Mlakar et al., 2003).

### 5.2.1 STRUCTURAL SYSTEM

The Pentagon is composed of five concentric rings (labelled A-E in Figure 100), which are divided into 10 sections (labelled 1-10 in Figure 10) (Mlakar et al., 2003).

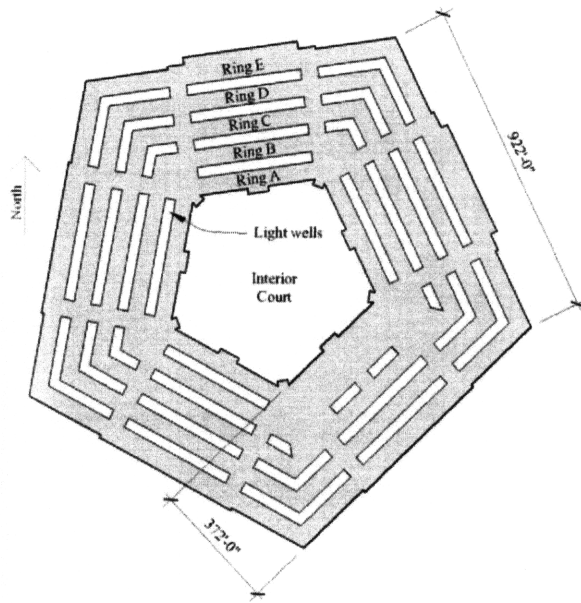


Figure 10: Pentagon Rings (Mlakar et al., 2003)

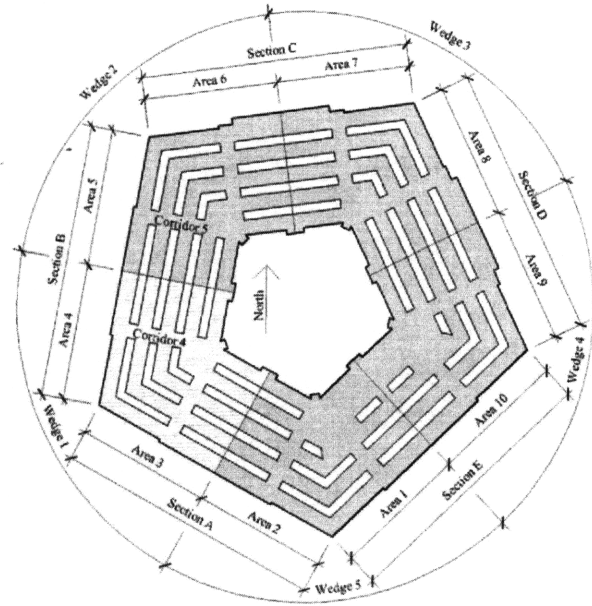


Figure 11: Pentagon Sections (Mlakar et al., 2003)

The entire structural system was constructed using cast-in-place, normal weight reinforced concrete, with a specified concrete strength of 2,500 psi and a yield strength of 40,000 psi in the reinforcing bars. The floor systems consist of slabs, beams, and girders resting on columns. Spiral reinforcement was used in nearly all columns supporting more than one story, while ties were used in the less loaded columns. The floor slabs are 5.5 in thick and span 10 ft between beams. The beams, in turn, span 10 to 20 ft between girders at their midspan. Finally, the girders span 20 ft between columns (Mlakar et al., 2003). Figure 12 depicts a typical floor layout, along with the typical beam and girder dimensions.

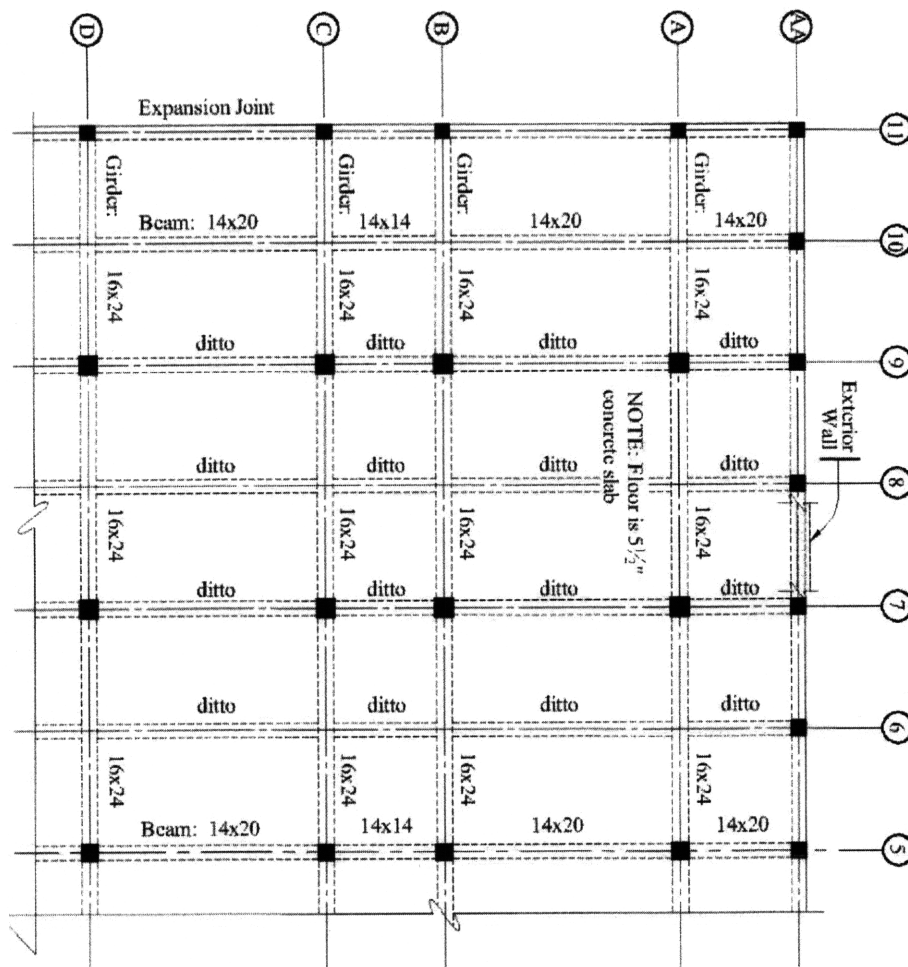


Figure 13: Typical Pentagon Floor Layout (Mlakar et al., 2003)

All exterior walls are 10 in thick reinforced concrete, with the exception of the outermost wall of Ring E, which consists of unreinforced brick infill and limestone facing. The concrete walls contain a grid of smaller columns and girders, which separate 5 by 7 ft windows (Mlakar et al., 2003).

The longitudinal reinforcement in the slabs, beams, and girders consists of straight and trussed bars,

similar to the beam shown in Figure 13.

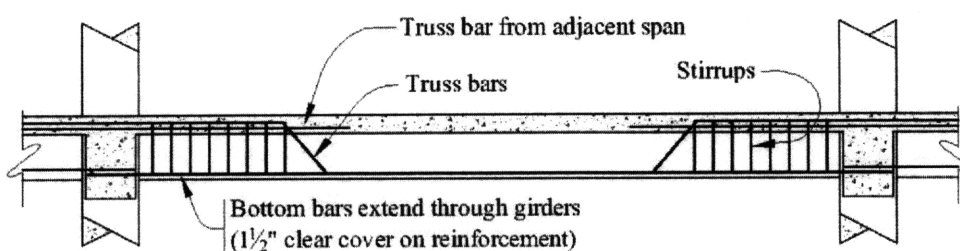


Figure 14: Typical Pentagon Beam Reinforcement Detail (Mlakar et al., 2003)

An important feature of the bottom longitudinal reinforcement in beams and girders is continuity at the supports. Of the bottom reinforcing bars in the beams and girders, approximately half of them extend through the supports, by laps of 30 to 40 times their diameter (Mlakar et al., 2003).

## 5.2.2 ACCIDENT INVESTIGATION AND STRUCTURAL RESPONSE

Figure 14 is a rendering of the west façade of the Pentagon and the position of the Boeing

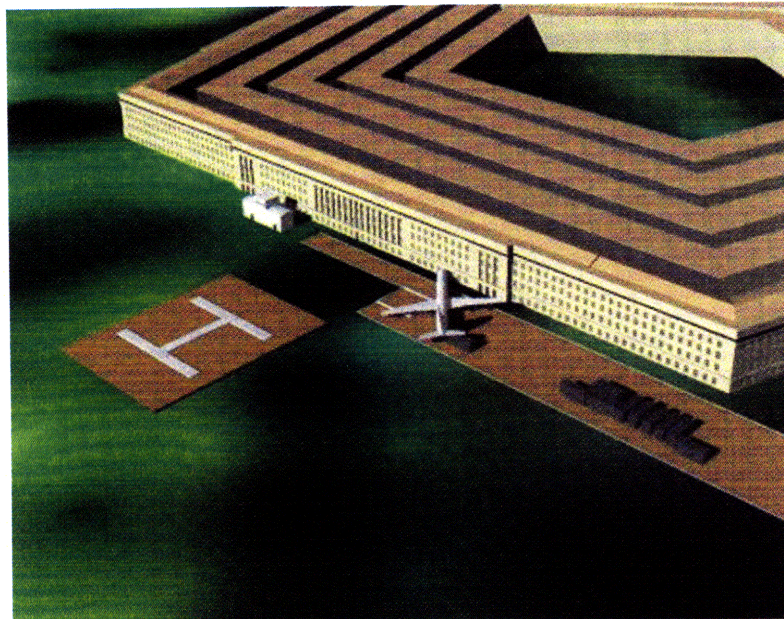


Figure 15: Location of Boeing 757 Crash  
(Mlakar et al., 2003)

757 as it first came into contact with the building. The extent of immediate damage due to impact was confined on the most part to the 1<sup>st</sup> story, where the aircraft struck the building. Most of the aircraft maintained a steady altitude between the ground floor slab and the second floor slab as it moved through the building; hence, the second story did not suffer much direct impact (Mlakar et al., 2003).

The *Pentagon Building Performance Report* (Mlakar et al., 2003) describes the effect of the moving fluid, consisting of jet fuel and solid debris, on the degree of damage to the structure upon impact. The columns along the impacted façade were immediately destroyed by the rushing fluid. In light of the high velocity at which the airplane struck the façade, the BPS concluded that the loss of these columns was inevitable. Adjacent columns that were not in the direct path of the aircraft lost their



concrete cover as they were impacted by fluid, but retained the capacity to absorb energy in flexure without failing. The BPS team attributed this to the use of spiral reinforcement as opposed to ties. Analyses of selected columns revealed that the spirally reinforced cores had a much higher calculated unit curvature capacity than if they had been tied columns. In addition, the shear corresponding to the maximum flexural capacity under lateral load was not greater than 75% of the shear strength of the columns. Thus, the capacity of the columns was governed by flexure rather than shear, allowing for a more ductile response to the blast loads. The adequate development of flexural strength in the cores was also a product of their anchorage length, which was sufficient to prevent the columns from suddenly tearing out of their supports. In those columns that did fail, the reinforcement underwent considerable necking prior to tearing off. Figure 16 is a photo of a column in the Pentagon that was torn off at the support due to impact. It can be observed that the ends of the longitudinal bars experienced necking, indicating ductile failure.

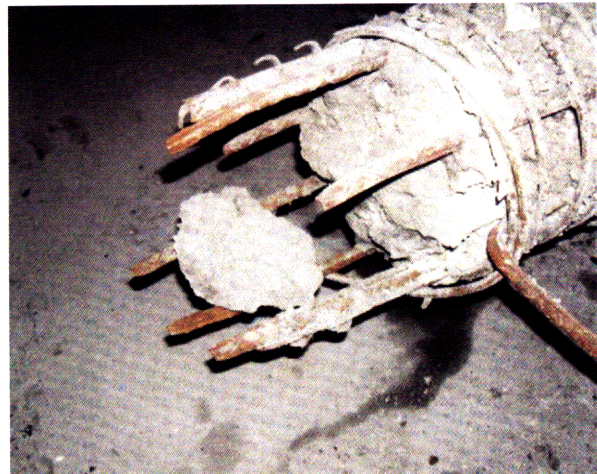


Figure 17: Necking of Column Rebar (Mlakar et al., 2003)

Figure 16 summarizes the response of the 1<sup>st</sup> story columns in the area impacted by the plane. The wedges shown in the drawing refer to the division of sections for the execution of the renovation project. The renovation of Wedge 1 was practically complete at the time of impact (Mlakar et al., 2003).

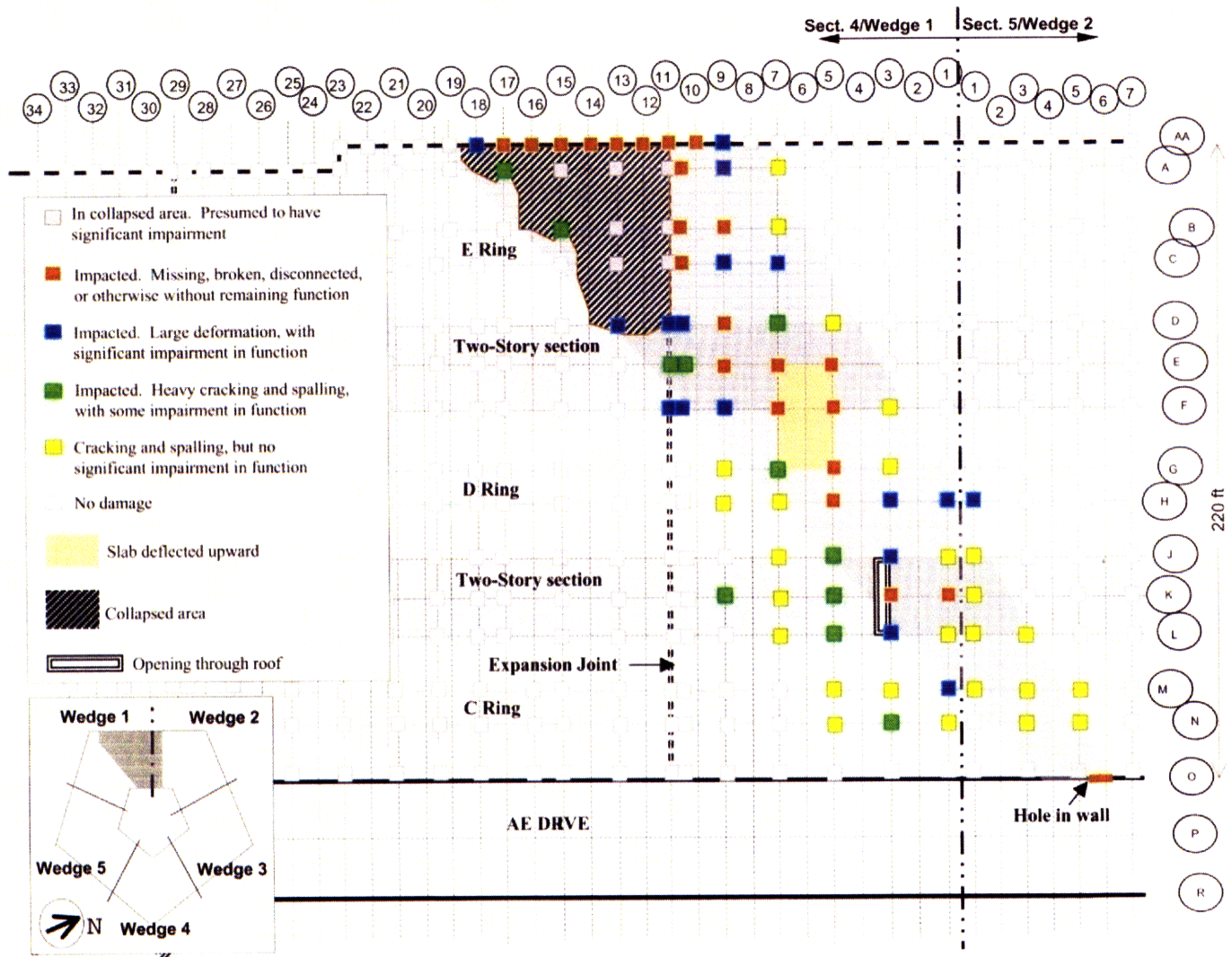


Figure 18: Summary of First Story Damage (Mlakar et al., 2003)

Despite the damage inflicted on the 1<sup>st</sup> story columns, most of the floor area on the 2<sup>nd</sup> story remained intact. This was predominantly due to structural redundancy, which can be observed in several forms in the Pentagon. The typical beam and girder layout, described earlier in this report



and presented in Figure 12, reveals that relatively short spans were used in design, particularly through the use of beams spanning between girders from midspan to midspan (Mlakar et al., 2003). This floor system provides extensive support to the slabs and makes it possible for alternative load paths to form in the event of column loss. The spans between columns are also sufficiently short that catenary action can be developed in the girders above if intermediate supports are lost.



Figure17: Load Transfer through Brick Façade  
(Mlakar et al., 2003)

girder supports is a key element of ductility that provides considerable protection against progressive collapse in the event of lost supports. The predominant use of lapped reinforcing bars through supports in the design of the Pentagon further enhanced the energy-absorbing capacity of the structure that remained after the initial impact, and thus contributed to the resistance of the affected area of the building to progressive collapse.

Additionally, it was observed following the September 11<sup>th</sup> disaster that the brick infill in the exterior walls behaved as transfer girders when columns were destroyed (Mlakar et al., 2003). Figure 19 depicts a similar phenomenon in a different building. With the aid of the exterior façade, the redistribution of gravity loads from the upper floors out to the side columns mitigated the further onset of progressive collapse.

As previously mentioned, the continuity of bottom reinforcement through beam and



Lastly, the live load used in the original design of the Pentagon was adjusted to include an additional 150 psf warehouse load (Mlakar et al., 2003). This reserve strength implies that the members were designed more heavily than needed for the intended occupancy of the building. This feature naturally increased the overall resistance of the structure to both the immediate and long-term effects of the September 11 crash.

It should be noted from Figure 16 that a small portion of Ring E between column lines 11 and 19 eventually experienced progressive collapse. However, the BPS team reported that the collapse did not occur until 20 minutes later, and that the cause of collapse was most likely overheating due to fire, combined with the additional pressure exerted by the water used to extinguish the fire. The loss of concrete cover at some locations may have further aggravated the susceptibility of the structure to fire damage. The results of yield line analyses and member capacity calculations suggested that, under normal temperatures, the floor segment between column lines 11 and 19 would have had the capacity to support itself and the floors above through the transfer of loads to adjacent segments (Mlakar et al., 2003).

### **5.3 COMPARATIVE ASSESSMENT**

A comparison between the structural properties of the Alfred Murrah Building and the Pentagon and their performance under blast loads helps reinforce the aforementioned ideas concerning blast-resistant design. In analyzing the features of both buildings, it becomes apparent that several of the design features that contributed to the resistance of the Pentagon to the effects of the September 11 crash were absent in the Murrah Building. Had the design of the Murrah Building incorporated some of these features, the response of the structure to the bomb blast might have been far less destructive.

Firstly, the design of the Murrah Building sacrificed shear walls and robust façade elements on the front face for the sake of providing more open access to the public. This approach was detrimental to the capacity of the structure to resist blast-induced impact loads and progressive collapse, as it lacked sacrificial elements and did not provide sufficient redundancy for load redistribution. Furthermore, the open configuration of the front façade created ample space for external vehicles to approach the building. The exterior structure of the Pentagon, on the other hand, proved to be highly resistant to blast by means of strong, brick-infilled exterior walls, along with glazed windows. Traffic barriers around the perimeter of the building also serve to protect it against infringement by external vehicles.

The reinforcement detailing of the Pentagon was also found to far surpass that of the Murrah Building in redundancy, ductility, and toughness. For instance, the efficiency of the spirally reinforced Pentagon columns in carrying extreme lateral and gravity loads was not as apparent in the tied columns of the Murrah Building. The lack of continuous bottom reinforcement through beam and girder supports also left the Murrah Building vulnerable to column loss and collapse. Moreover, the beam-to-column connections were not sufficiently ductile to absorb the additional energy from the plastically deformed beams and girders. Conversely, the structural components of the Pentagon were on the most part able to accommodate the redistribution of loads due to lost supports.

The design of the Pentagon was made even more resistant to progressive collapse by the added redundancy of a two-way floor system with relatively short spans, as well as the provision of a warehouse live load in excess of the prescribed occupancy load. When the building was subjected to a large and unforeseen impact load, the overdesign of the structural system proved highly valuable.

## **6. IMPLEMENTATION OF GUIDELINES AND CODES FOR BLAST RESISTANCE**

Blast resistance has become increasingly significant in the consideration and implementation of new guidelines for structural design. Past events involving terrorist bomb explosions and accidental blasts in buildings have given rise to extensive research and new developments in blast resistant design. While great progress has been made in the establishment of new design guidelines, especially within the government sector, general guidelines and codes have yet to be validated and implemented for more widespread applicability.

United States government agencies have established new sets of guidelines for the design of federal buildings for more effective resistance to blasts. In 2003, The General Services Administration (GSA) published a document entitled *Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings*, which contains comprehensive procedures for analysis and design of federal buildings to resist progressive collapse, predominantly as a result of blast damage (Gurley, 2008).

One of the greatest difficulties involved in developing standard procedures for blast resistant design is the classification of buildings according to risk level. The existence of guidelines such as those published by GSA does not preclude the need to quantify blast related risks for each design case, in order to determine an adequate level of blast resistance. In-depth risk assessments should not only account for the susceptibility of a building to explosions, but should also consider the function and occupancy of the building. Furthermore, a great deal of uncertainty exists with regard to the nature and intensity of blasts, thus adding to the challenge of establishing design codes based on set importance factors to account for blast.

A potentially effective way to address this problem is to devise a parametric system whereby the factors of safety for the design of a given building are variable upon the susceptibility of the building

to blast. Risk analysis results would dictate the likelihood that the building would experience blast loads in a given period of time, thus leading to more accurate modification factors for design. These modification factors could then be applied to the key elements of the structure, those which govern the overall resistance of the building to progressive collapse.

In the case of federal buildings, the government has well-defined procedures do establish blast threat levels. Once the building is assigned to a threat category, the design detailing is then prescribed specifically to address this potential threat. The three levels defined by federal guidelines to rate the level of threat of terrorist attacks alone are as follows (Paczak et al., 2005).

- i. High Threat Level: a high threat of attack has been verified. This threat level is typically assigned to highly important buildings, whose loss would have severe consequences, or to buildings that serve as cultural icons.
- ii. Medium Threat Level: a threat of attack has been verified. This threat level applies to buildings whose loss would have strong consequences on governing powers, or to buildings that serve as regional symbols.
- iii. Low Threat Level: a threat of attack is suspected. This threat level is assigned to buildings whose loss would have moderate consequences, or to buildings that might be regional symbols.

Non-federal buildings may also be at risk of terrorist attack or accidental blast, and may require special consideration for blast resistance. The lack of defined criteria for the assessment of threat levels makes it more difficult for structural engineers and project owners to decide upon appropriate design features to implement. Thus, if there exists any suspicion of a blast threat for a given building, the parties involved may hire a blast consultant. In designing a building for resistance to a potential blast, the consultant would not only implement existing guidelines for increased robustness

and ductility, but they would also be capable of conducting specialized analyses on the structure to test possible scenarios involving structural loss. Following a risk assessment of the building, the consultant could propose appropriate safety factors to account for blast effects in the analysis of specified loads. For instance, in order to account for floor rebound following an upwards blast, federal guidelines stipulate a load combination of  $2(DL) + 0.5(LL)$  for a medium-level threat (Paczak et al., 2005). The doubled dead load accounts for the possibility of a high downwards acceleration in the floor slab following uplift, as well as the potential loss of a midspan support. Similar factors can be applied to non-federal buildings and varied depending on perceived threat levels.

## 7. CONCLUSION

Blast-induced loads on buildings are gaining progressively more importance in design considerations, as a result of increasingly frequent and severe terrorist attacks as well as concerns with accidental explosions. The impact of blast shockwaves on buildings and the ensuing risk of structural loss and collapse render blast loading extremely hazardous to the integrity and safety of buildings. In order to mitigate this potential hazard, a number of techniques can be applied to the design and retrofit of buildings for resistance to blast loads. The most important aim of blast-resistant design is to ensure that buildings are not at risk of progressive collapse, thereby permitting the safe and timely egress of all occupants in the event of an explosion.

The effective design and retrofit of buildings for blast resistance should not only involve analysis and design methods for structural systems, but should also incorporate external features such as façade elements and barriers. The cheapest and most effective way to mitigate blast effects is to hinder the entry of explosives to the building or its vicinity. However feasible this technique may be, it might not be possible to guarantee the imperviousness of such exterior barriers to explosives. In order to account for this uncertainty, buildings that are at a higher risk of blast should also be designed to resist powerful blast loads. Two of the most essential structural design considerations are redundancy and ductility. Improved ductility in members and connections allows for higher levels of energy absorption through plastic deformation, thus avoiding sudden failures. In order to account for the possibility of some structural loss, it is also crucial that the structure be designed for redundant load paths.

Through the aforementioned case studies of two well known blast occurrences, the Oklahoma City bombing and the attack on the Pentagon on September 11, several examples of design and retrofit

applications for blast resistance have been analyzed. Investigative reports on both disasters have permitted further studies into effective techniques for blast resistance, and a comparison of both cases reveals the structural weaknesses that contribute to the susceptibility of a building to blast, as well as the features that enhance the capacity of a structure to absorb energy from a blast and effectively redistribute loads. Through the investigation of such occurrences, it becomes possible to make further developments in blast resistant design, in an ongoing effort to produce safe and efficient structural systems based on the unique threat level of each building.

While great progress has been made in developing building elements and systems for enhanced blast resistance, particularly in federal buildings, a lingering challenge of blast resistant design is the difficulty involved in standardizing codes and guidelines. The high level of uncertainty involved in blast loading, combined with the disparate levels of threat associated with different buildings, gives rise to significant obstacles in the provision of importance and safety factors for design. Therefore, as a means to circumvent this limitation, it is important that comprehensive risk analysis expertise be applied to the analysis and design of individual buildings, such that their structural and architectural systems will resist blast-induced progressive collapse with optimal efficiency.

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